LOCAL BUCKLING OF COLD FORMED STEEL IN COMPOSITE STRUCTURAL ELEMENTS AT ELEVATED TEMPERATURES

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UNICIV REPORT No. R-327 JANUARY 1994
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KENSINGTON NSW AUSTRALIA 2033

ISBN : 85841 294 2
An inelastic semi-analytical finite strip method is used to analyse the local buckling behaviour of cold formed steel plates of composite steel-concrete structural elements at elevated temperatures. The decrease in yield stress and elastic modulus of the cold formed steel is represented at elevated temperatures by a series of nonlinear stress-strain relationships. The change in the material properties at elevated temperatures is shown to influence the local buckling behaviour of steel plate elements in steel-concrete composite construction. Various boundary conditions are chosen in a parametric study, in which the influence of different variables is determined. Recommendations are given for the assignment of slenderness limits, and a simple method is presented to obtain the local buckling stresses at elevated temperatures. A design example of a profiled composite beam is used to illustrate the method presented.
1. INTRODUCTION

The reduction in strength and stiffness of steel at elevated temperatures is detrimental to steel encased concrete composite structures. The reduction in strength usually therefore requires an adequate fire resistant material to be supplied, so that a reasonable fire resistance time may be achieved. Composite structures that fall into this category are concrete filled rectangular and tubular steel columns which are usually of mild structural steel [1,2]. A newly developed form of composite structure involves the use of cold formed steel used in composite slabs [3,4], and more recently in profiled composite walls [5] and in profiled composite beams [6,7].

Composite elements such as concrete filled tubes usually are unreinforced, and thus when subjected to fire the concrete core must carry the load. The relevant international standards for building and design outline the reduced design loads required for the important fire limit state.

The reduced loads required to be carried for the fire limit state often require that only small amounts of nominal reinforcement should be included. However, this is based on the assumption that the steel is ineffective in terms of strength. Fire limit states also assume that the structure will be inadequate for routine use after the event of a fire. This practice is conservative, and it does not provide a true understanding of the structural behaviour and it is also uneconomical. The economical savings in using a composite structural element may be substantially eliminated if fire protective coatings are required to achieve specified fire durations, as outlined in various national codes of practice.

The emergence of a large variety of structural elements with exposed steel has recently inspired researchers to evaluate the contribution of the steel to structural strength at elevated temperatures. This therefore requires that the structure be designed under various load combinations, as is routine in limit states or load and resistance factor design. Consideration of a composite element at elevated temperatures such as may be experienced in a fire is therefore a necessary requirement in performing a limit states analysis [8].

In the interest of restoration and economy, a composite steel-concrete element should be designed to be able to maintain its strength throughout severe temperatures, or its behaviour should be at least clearly understood. The purpose of this study is to address the failure mode of local buckling of steel plates in composite structural elements when subjected to elevated temperatures.
This study has been undertaken for the purpose of identifying the behaviour of cold formed steel in profiled composite beams. The constitutive relationship of cold formed steel has been used, however the study is not strictly germane to the area of profiled composite construction where cold formed steel is used. The elastic regions of the curves developed in this paper may be used for any value of yield stress, with an appropriate modification. Thus the analysis presented may be used for any composite construction method where a rigid medium (such as concrete) restrains the steel from buckling at elevated temperatures.

The analysis of structural elements experiencing elevated temperatures, such as those due to a fire, are outlined in various international building codes. The Australian Standards AS1170.1 for dead and live loads [9], AS 3600 for concrete [10] and AS 4100 for steel [11], require that a structure or structural element be designed to be satisfactory against the following load combination:

\[ w^* = 1.1G + 0.4Q \]  

(1)

where \( w^* \) represents the total design or factored load, \( G \) is the dead load and \( Q \) is the applied live load. On the other hand, the relevant British Standard for the design of composite structures for fire, BS5950: Part 8 [12] requires the structure to be designed to carry the load combination

\[ w^* = 1.0G + 1.0Q \]  

(2)

where \( w^* \), \( G \) and \( Q \) are the same loads as above. Other national standards present similar load combinations.

Olawale and Plank [13] studied the collapse analysis of steel columns in fire using an inelastic finite strip method developed by Mahmoud [14]. The study undertaken by Olawale and Plank considered the loaded edges of the finite strips as simply supported. This study was further documented by Burgess et al. [15]. Burgess et al. [16] studied the reduced stiffness of steel beams when subjected to fire, and developed the moment curvature response. All of the above studies involved modelling the degradation of structural steel properties of the yield strength and elastic modulus.
The analysis of concrete structures at elevated temperatures also involves modelling the degradation of the material properties. The material properties that are of primary interest in concrete analysis are the compressive strength and the elastic modulus. Values for these properties at different temperatures are given in the Australian AS 3600 [10] and by Warner et al. [17]. Recent research by Khoury [18] also shows the influence of elevated temperatures on the compressive strength of the concrete.

Schmidt and Lehmann [19] undertook tests on the fire resistance of composite deck slabs. Various rib configurations were used, and a very extensive test series was carried out to ascertain the period of time over which the profiled composite slabs could resist the applied design fire load. A similar series of tests was undertaken by Cook et al. [20] for the fire behaviour of profiled steel sheet floors. Simply supported and continuous composite slabs were tested in both of the above studies. These studies used a furnace to heat the specimens using the standard fire curve as outlined in various technical publications [21].

Concrete-filled steel tubes are used extensively in high-rise buildings, and so composite column behaviour at elevated temperatures has been of interest for some time. Amongst more recent studies, Matsumara and Sakuma [22] undertook an experimental study of the fire resistance of concrete filled square tubular steel columns without fire protection.

Other notable studies of composite structures at elevated temperatures include those by Lie and Chabot [23], who studied the fire resistance of circular concrete filled hollow steel columns by conducting full scale fire tests. Lie and Chabot [24] then provided mathematical representations of these compression members subjected to fire.

An extensive numerical simulation based on a finite element study was conducted by Schleich et al. [25] on composite structural elements or frames. This study was calibrated quite closely with existing fire tests. The study suggested that the computer simulation could not completely substitute for real fire tests, due to local problems occurring such as spalling of concrete, bond failure of reinforcing with concrete and also the local buckling of steel. Computer simulation is important, though, since real fire tests invariably are very expensive.

This study is aimed at developing some understanding and design data in the area of the local buckling of thin steel sheeting, and its occurrence at elevated temperatures, which cannot be achieved economically in practice because of the prohibitive cost of fire testing. This will be a helpful supplement to all the previous studies of the composite structures
mentioned, since the results can be applied to any member in compression, bending or compression and bending. The method of analysis used herein is based on the well-known finite strip method of buckling analysis.

2. FINITE STRIP METHOD

2.1 General

The method used for the inelastic local buckling analysis is the semi-analytical finite strip method presented by Cheung [26]. The strip used is a lower order strip element with two nodal lines and four degrees of freedom per nodal line. The method has been well-documented, and only a brief summary, as it applies to cold formed steel in composite elements, is presented here.

The finite strip method uses a harmonic series function in the longitudinal direction and a polynomial in the transverse direction to describe the displacements of a plated structure. The strip assumes that there is no shear within the local buckle half-wavelength, which is a reasonable assumption for small values of half-wavelength as are experienced in local buckling of steel in contact with a rigid medium [27].

2.2 Displacements

For the particular case of the buckling of plates which are by restrained or in contact with a rigid medium, the finite strip must satisfy the conditions of both zero deflection and zero slope at the two ends, as shown in Fig. 1. This strip has been fully developed previously by the authors [28], using a sine squared displacement function in the longitudinal direction for both bending and membrane displacements. A cubic and linear polynomial are used to describe the transverse displacements for the bending and membrane actions respectively.
2.3 Inelastic local buckling

The study undertaken by the authors [28] considers the elastic local buckling of thin steel plates in composite elements. The extension of this study has been applied to the inelastic range of structural response, by taking into account the material non-linearity of the cold formed steel [29].

Material properties

In order to describe the stress-strain relationship at elevated temperatures, the constitutive material properties are required. Olawale and Plank [13] used the draft version of the British BS 5950: Part 8 [30] to describe the elastic modulus and yield stress of mild structural steel at elevated temperatures.

The behaviour of cold formed steel at elevated temperatures is not covered by the Australian Standard for cold formed steel structures, AS1538 [31], which does not outline any requirements for the design during fire. The British Standard BS5950: Part 8 [12] has tabulated values for the strength reduction of cold formed steels at elevated temperatures. These were used herein to obtain the values of yield stress at certain temperatures. The BS5950: Part 8 does not, however, outline values for the reduction in the elastic modulus with elevated temperature. The Australian Standard AS4100 [11] for hot-rolled structures does outline values for yield stress and elastic modulus at elevated temperatures. Since the elastic modulus values for cold formed steels and mild structural steels at ambient temperatures are close, these were adopted for the ensuing analysis. The reductions in yield stress and elastic modulus at elevated temperatures are shown in Figs. 2 and 3 respectively.

2.5 Stress-strain relationship

A modification to the inelastic finite strip method presented by Uy and Bradford [29] is needed to undertake a fire analysis. This modification requires the relevant stress-strain behaviour to be described at discrete values of temperature. A modification of the well-cited Ramberg and Osgood representation of stress and strain [32] was presented by Olawale and Plank [13] for the modelling of mild structural steel at elevated temperatures. This expression describes the strain $\varepsilon_i$ in the steel as a continuous function of the stress $\sigma_i$, and is given by
\[ \varepsilon_i = \frac{\sigma_i}{E} + \frac{3}{7} \left( \frac{\sigma_y}{E} \right) \left( \frac{\sigma_i}{\sigma_y} \right)^n \]  

where \( n \) is a parameter that describes the shape of the knee of the stress-strain relationship, typically taken as 25 for cold formed steels. The stress-strain relationships at different temperatures for two different cases are shown in Figs 4 and 5.

3. RESULTS

3.1 Verification of model

To verify the model, the accuracy of the elastic finite strip method has been established by the authors [29], with various boundary conditions being considered. The inelastic finite strip method is an extension of this analysis, incorporating material non-linearity, which therefore requires an alternative solution method as described by Uy and Bradford [29]. For simply supported strips, this was established by Azhari and Bradford [33]. The study herein was applied to cold formed steel, however as discussed previously, different yield stresses can be accounted for in the elastic range, where the slenderness limits are proportional to the square root of the yield stress [8].

3.2 Parametric study

A parametric study has been undertaken to study the local buckling behaviour of cold formed profiled steel sheets at elevated temperatures. This has been applied to various boundary conditions and different strain gradients. The method is useful for ascertaining the local buckling stress or strains of composite structures at elevated temperatures. The following boundary conditions were considered, with all cases having clamped loaded sides:

(i) two clamped unloaded sides;
(ii) one simply supported and one clamped unloaded side;
(iii) two simply supported unloaded sides;
(iv) one clamped and one free unloaded side; and
(v) one simply supported and one free unloaded side.

The parametric study was applied to various temperatures from ambient, \( T = 20^\circ\text{C} \) to a temperature of \( T = 600^\circ\text{C} \). The yield stress and elastic modulus at ambient temperatures
were taken as 550 N/mm$^2$ and 200,000 N/mm$^2$ respectively, these being representative of cold formed steel. The assumption made is that the temperature of the steel throughout the plate width and thickness is constant, which is a fairly accurate assumption for steel in composite steel-concrete structural elements [25].

The inelastic local buckling analysis was undertaken for temperatures of the following values, $T = 20, 200, 300, 400, 500$ and $600^\circ$C. The strain gradient was varied between uniform compression, $\alpha = 1.0$ and triangular compression, $\alpha = 0.0$. The results of the analysis showing the dimensionless critical stress against slenderness are shown in Figs. 6 and 7 for two clamped unloaded edges. The dimensionless critical strains have also been plotted against slenderness, and these are given later in Figs. 16 and 17.

The study was then applied to a plate with three clamped loaded sides with one unloaded edge being simply supported. The same temperature values were chosen, and the results of critical stresses for various slendernesses are given in Figs. 8 and 9 for a strain gradient of $\alpha = 1.0$ and $\alpha = 0.0$ respectively.

Figures 10 and 11 show the results for the above studies, but for two simply supported unloaded edges. A study was also undertaken of a plate with one clamped and one free unloaded side. The results of this study are shown in Figs. 12 and 13. These curves show that except for slenderness values of less than 30, elastic local buckling will occur. Finally, the case of one simply supported and one free unloaded side was considered. This study also ascertained that except for very small slendernesses, the local buckling will be elastic. The results for this final case are given in Figs. 14 and 15.

The direct relationships required to obtain local buckling stresses at any temperature $T$ is

\[
\text{for } \sigma_{\alpha\ell}(T) \leq f_{yp}(T); \quad \sigma_{\alpha\ell}(T) = \sigma_{\alpha\ell}(20) \cdot \frac{E(T)}{E(20)}
\]  

\[\text{for } \sigma_{\alpha\ell}(T) > f_{yp}(T); \quad \sigma_{\alpha\ell}(T) = \sigma_{\alpha\ell}(20) \cdot \frac{f_{yp}(T)}{f_{yp}(20)}\] 

in the elastic range and

in the inelastic range. Thus a trial and error procedure must be deployed to determine whether buckling is either elastic or inelastic.
The elastic local buckling stress $\sigma_{el}$ at ambient temperature ($T = 20^\circ C$) is given by the equation

$$\sigma_{el}(20) = \frac{k \pi^2 E}{12 (1-\nu^2) \left( \frac{b}{t} \right)^2}$$

The values of the elastic local buckling coefficients $k$ have been derived by Uy and Bradford [29] for all the boundary conditions used here. These can also be calculated directly from the curves derived within this study. The inelastic local buckling stress at ambient temperature ($T = 20^\circ C$) can be obtained from the inelastic local buckling analysis undertaken by Uy and Bradford [29].

The British Standard BS5950: Part 8 [12] suggests that for profiled composite slabs during a fire the plastic moment of the slab only may be assumed. This practice is conservative if the reduced value of the yield strength and elastic modulus is used at the temperature it is designed to withstand. The practice, however, is not applicable to the design of continuous slab over the negative moment region, because local instability may occur. The method presented here can account for this, and the application of a post buckling model can be used to develop the full behaviour.

4. DESIGN APPLICATION

A design application is given here to demonstrate the use of the model when applied to a composite structural element. A profiled composite beam is used, which is a new form of construction method for reinforced concrete beams [7, 34].
Problem:

Given the profiled composite cross-section shown in Fig. 18 calculate the values of local buckling stress of the steel at the top plate in compression at elevated temperatures that can be used for design purposes in calculating the strength of the cross-section.

Results:

The results can be obtained by using the curves of Figs. 6 and 7 to produce the buckling stresses. The two values of strain gradient $\alpha = 1.0$ and $\alpha = 0.0$ are represented in Figs. 6 and 7 respectively. Linear interpolation can be used to obtain values of strain gradient which are intermediate to those given [29]. The neutral axis depth can be determined from a cross-sectional analysis, and the value of strain gradient, $\alpha$, can then be calculated.

The values that can be used in design for a slenderness of $\frac{b}{t} = 100$ are tabulated in Table 1.

Discussion:

The values of the stresses presented in Table 1 can then be used in ascertaining the onset of local buckling. A postbuckling model, once developed, can then be used to calculate the reduced stiffness of the composite element with increasing load. This will then provide a full-range understanding of the behaviour of the composite structural element.

The method presented in this design example can also be applied to other structural elements in which steel is restrained from buckling by a rigid medium. Some applicable composite construction applications are shown in Figure 19. These include the compression flange in composite beams under positive bending, and composite columns or profiled composite walls under compression or combined bending and compression. Other applications include profiled composite slabs in negative moment regions.
5. CONCLUSIONS

From the results presented herein, it is evident that the reduction in the local buckling stress can be quantified by an inelastic load buckling analysis for ambient temperature, and by taking into account the reduction in elastic modulus and yield strength at elevated temperatures, the local buckling stresses can be obtained directly. This is quite useful and therefore reduces the amount of analysis required.

The study has presented a number of major items which pertain to the behaviour of steel at elevated temperatures. Firstly the stress-strain behaviour has been developed for various elevated temperatures. The second item has been the determination of the stresses required to cause local buckling of the steel at elevated temperatures in composite elements, and the strains achieved at local buckling. A design application has then been given which demonstrated the application of the results to a practical situation.

The important but somewhat paradoxical point obtained from this research is that the required slenderness limits at elevated temperatures are much higher to avoid local buckling than those that exist at ambient temperature. Therefore if the plates in composite elements are designed against local buckling at ambient temperatures, then the design will be adequate under fire conditions. This is because of the variation of the yield stress and elastic modulus, as well as the local buckling stress, with temperature.

Further research into composite structural elements can be advanced by undertaking the following:

(i) Ascertain accurately by tests the yield strength and elastic modulus for cold formed and structural steels at elevated temperature to be used as input for numerical models;

(ii) Full scale experimental fire tests for the development of new construction methods, such as profiled composite beams, to provide data for calibration of numerical models;

(iii) Development of a numerical model to describe the behaviour of a member at elevated temperature and over time; and

(iv) The development of the postbuckled behaviour of plates in contact with a rigid medium.
6. ACKNOWLEDGEMENTS

This paper forms part of a programme into the strength and serviceability of composite steel-concrete members being undertaken at The University of New South Wales. The profiled composite beam programme is funded by a large Australian Research Council Grant. The first author was supported by an Australian Postgraduate Research Award and a Faculty of Engineering, Dean's Scholarship.

REFERENCES


**PRINCIPAL NOTATION**

\( b \) width of plate

\( E \) Young's modulus of steel

\( f_{yp} \) yield stress of profiled steel sheeting (in N/mm²)

\( G \) dead load

\( k \) buckling coefficient

\( L \) buckling half-wavelength

\( Q \) live load

\( t \) thickness of plate

\( T \) temperature

\( w^* \) total design load

\( \alpha \) stress gradient

\( \varepsilon \) strain

\( \varepsilon_{ol} \) local buckling strain

\( v \) Poisson's ratio

\( \sigma \) stress

\( \sigma_{ol} \) local buckling stress

\( \sigma_y \) yield stress
<table>
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<tr>
<th>Steel Temperature (°C)</th>
<th>$\frac{\sigma_{ol}}{f_{yp}(20)}$</th>
<th>$\alpha = 1.0$</th>
<th>$\alpha = 0.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.62</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0.57</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>400</td>
<td>0.55</td>
<td>0.29</td>
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<tr>
<td>500</td>
<td>0.44</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>0.31</td>
<td>0.17</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Local buckling stresses at elevated temperatures for profiled composite beam example
Figure 1. Geometry of thin strip.
Figure 2. Yield strength reduction factors for cold formed steels.
Figure 3. Elastic modulus reduction factors.

\[
\frac{E(T)}{E(20)} = 1.0 + \left\{ \frac{T}{2000 \ln(\frac{T}{1150})} \right\}
\]
when \( 0^\circ C < T \leq 600^\circ C \)

\[
\frac{E(T)}{E(20)} = \frac{690(1 - \frac{T}{1000})}{T - 53.5}
\]
when \( 600^\circ C < T \leq 1000^\circ C \)
Figure 4. Stress–strain curves for elevated temperatures at 2.0 % strain limit
Figure 5. Stress–strain curves for elevated temperatures at 0.5 % strain limit
Figure 6. Dimensionless critical stress vs slenderness ($\alpha=1.0$)
Figure 7. Dimensionless critical stress vs slenderness ($\alpha=0$)
Figure 8. Dimensionless critical stress vs slenderness ($\alpha=1.0$)
Figure 9. Dimensionless critical stress vs slenderness ($\alpha=0$)
Figure 11. Dimensionless critical stress vs slenderness ($\alpha=0$)

$\frac{\sigma_{cr}}{f_{yp}(20)}$

- C - clamped
- S - simply supported

$f_{yp}(20) = 550 \, N/mm^2$

$E(20) = 200 \times 10^3 \, N/mm^2$

2.0% strain limit

$T=20^\circ C$

$T=200^\circ C$

$T=300^\circ C$

$T=400^\circ C$

$T=500^\circ C$

$T=600^\circ C$
Figure 12. Dimensionless critical stress vs slenderness ($\alpha=1.0$)

\[ \frac{\sigma_{cl}}{f_{yp}(20)} \]

- $C$ - clamped
- $S$ - simply supported

$f_{yp}(20) = 550 \text{ N/mm}^2$

$E(20) = 200 \times 10^3 \text{ N/mm}^2$

2.0% strain limit
Figure 13. Dimensionless critical stress vs slenderness ($\alpha=0$)
Figure 14. Dimensionless critical stress vs slenderness ($\alpha=1.0$)
Figure 15. Dimensionless critical stress vs slenderness ($\alpha=0$)
Figure 16. Dimensionless critical strain vs slenderness (α=1.0)
Figure 17. Dimensionless critical strain vs slenderness (α=0)
(a) Profiled composite beam

\[ E = 200,000 \text{ N/mm}^2 \]
\[ \nu = 0.30 \]
\[ f_{yp} = 550 \text{ N/mm}^2 \]
\[ t = 1.0 \text{ mm} \]

(b) Boundary condition of uppermost plate

[Diagram showing stress distribution and idealised boundary condition of uppermost plate]

Figure 18. Profiled composite beam example
Figure 19. Composite construction applications